

CHAPTER 5

STORM DRAINS AND CULVERTS

5-1. General.

The storm-drain system should have sufficient capacity to convey runoff from the design storm within the barrel of the conduit. Hydraulic design of the storm-drain system is discussed in TM 5-820-4/AFM 88-5 chapter 4. A drainage culvert is a relatively short conduit used to convey flow through a roadway embankment or past some other type of flow obstruction. Culverts are constructed from a variety of materials and are available in many different shapes and configurations. Culvert hydraulics and diagrams, charts, coefficients, and related information useful in design of culverts are shown in TM 5-820-4/AFM 88-5 chapter 4.

5-2. Headwalls and endwalls.

a. The normal functions of a headwall or wing-wall are to recess the inflow or outflow end of the culvert barrel into the, fill slope to improve entrance flow conditions, to anchor the pipe and to prevent disjoining caused by excessive pressures, to control erosion and scour resulting from excessive velocities and turbulences, and to prevent adjacent soil from sloughing into the waterway opening.

b. Headwalls are particularly desirable as a cutoff to prevent saturation sloughing, piping, and erosion of the embankment. Provisions for drainage should be made over the center of the head-wall to prevent scouring along the sides of the walls.

c. Whether or not a headwall is desirable depends on the expected flow conditions and embankment stability. Erosion protection such as riprap or sacked concrete with a sand-cement ratio of 9:1 may be required around the culvert entrance if a headwall is not used.

d. In the design of headwalls some degree of entrance improvement should always be considered.

The most efficient entrances would incorporate one or more of such geometric features as elliptical arcs, circular arcs, tapers, and parabolic drop-down curves. Elaborate inlet design for a culvert would be justifiable only in unusual circumstances. The rounding or beveling of the entrance in almost any way will increase the culvert capacity for every design condition. These types of improvements provide a reduction in the loss of energy at the entrance for little or no additional cost.

e. Entrance structures (headwalls and wingwalls) protect the embankment from erosion and, if properly designed, may improve the hydraulic characteristics of the culvert. The height of these structures should be kept to the minimum that is consistent with hydraulic, geometric, and structural requirements. Several entrance structures are shown in figure 5-1. Straight headwalls (fig 5-1a) are used for low to moderate approach velocity, light drift (small floating debris), broad or undefined approach channels, or small defined channels entering culverts with little change in alignment. The "L" headwall (fig 5-1b) is used if an abrupt change in flow direction is necessary with low to moderate velocities. Winged headwalls (fig 5-1c) are used for channels with moderate velocity and medium floating debris. Wingwalls are most effective when set flush with the edges of the culvert barrel, aligned with stream axis (fig 5-1d) and placed at a flare angle of 18 to 45 degrees. Warped wingwalls (not shown) are used for well-defined channels with high-velocity flow and a free water surface. They are used primarily with box culverts. Warped headwalls are hydraulically efficient because they form a gradual transition from a trapezoidal channel to the barrel. The use of a drop-down apron in conjunction with these wingwalls may be particularly advantageous.

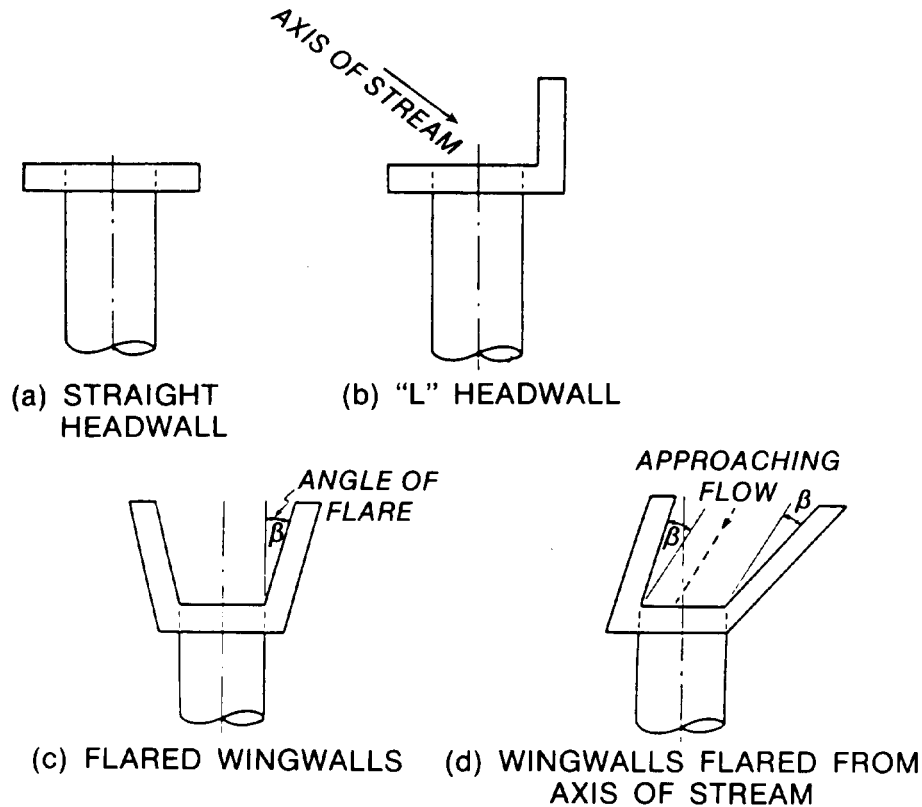


Figure 5-1. Culvert headwalls and wingwalls.

f. Headwalls are normally constructed of plain or reinforced concrete or of masonry and usually consist of either a straight headwall or a headwall with wingwalls, apron, and cutoff wall, as required by local conditions. Definite design criteria applicable to all conditions cannot be formulated, but the following comments highlight features which require careful consideration to ensure an efficient headwall structure.

(1) Most culverts outfall into a waterway of relatively large cross section; only moderate tailwater is present, and except for local acceleration, if the culvert effluent freely drops, the downstream velocities gradually diminish. In such situations the primary problem is not one of hydraulics but is usually the protection of the outfall against undermining bottom scour, damaging lateral erosion, and perhaps degrading the downstream channel. The presence of tailwater higher than the culvert crown will affect the culvert performance and may possibly require protection of the adjacent embankment against wave or eddy scour. In any event, a determination must be made about downstream control, its relative permanence, and tailwater conditions likely to result. Endwalls (outfall headwalls) and wingwalls will not be used

unless justifiable as an integral part of outfall energy dissipators or erosion protection works, or for reasons such as right-of-way restrictions and occasionally aesthetics.

(2) The system will fail if there is inadequate endwall protection. Normally the end sections may be damaged first, thus causing flow obstruction and progressive undercutting during high runoff periods which will cause washout of the structure. For corrugated metal (pipe or arch) culvert installations, the use of prefabricated end sections may prove desirable and economically feasible. When a metal culvert outfall projects from an embankment fill at a substantial height above natural ground, either a cantilevered free outfall pipe or a pipe downspout will probably be required. In either case the need for additional erosion protection requires consideration.

g. Headwalls and endwalls incorporating various designs of energy dissipators, flared transitions, and erosion protection for culvert outfalls are discussed in detail in subsequent sections of this chapter.

h. Headwalls or endwalls will be adequate to withstand soil and hydrostatic pressures. In areas of seasonal freezing the structure will also be designed to preclude detrimental heave or lateral

displacement caused by frost action. The most satisfactory method of preventing such damage is to restrict frost penetration beneath and behind the wall to nonfrost-susceptible materials. Positive drainage behind the wall is also essential. Foundation requirements will be determined in accordance with procedures outlined in note 4 of paragraph 2-6d. Criteria for determining the depth of backfill behind walls are given in TM 5-818-1.

i. The headwalls or endwalls will be large enough to preclude the partial or complete stoppage of the drain by sloughing of the adjacent soil. This can best be accomplished by a straight headwall or by wingwalls. Typical erosion problems result from uncontrolled local inflow around the endwalls. The recommended preventive for this type of failure is the construction of a berm behind

the endwall (outfall headwall) to intercept local inflow and direct it properly to protected outlets such as field inlets and paved or sodded chutes that will conduct the water into the outfall channel. The proper use of solid sodding will often provide adequate headwall and channel protection.

5-3. Scour at outlets.

In general, two types of channel instability can develop downstream from storm sewer and culvert outlets, i.e., either gully scour or localized erosion termed a scour hole. Distinction between the two conditions can be made by comparing the original or existing slope of the channel or drainage basin downstream of the outlet relative to that required for stability as illustrated in figure 5-2.

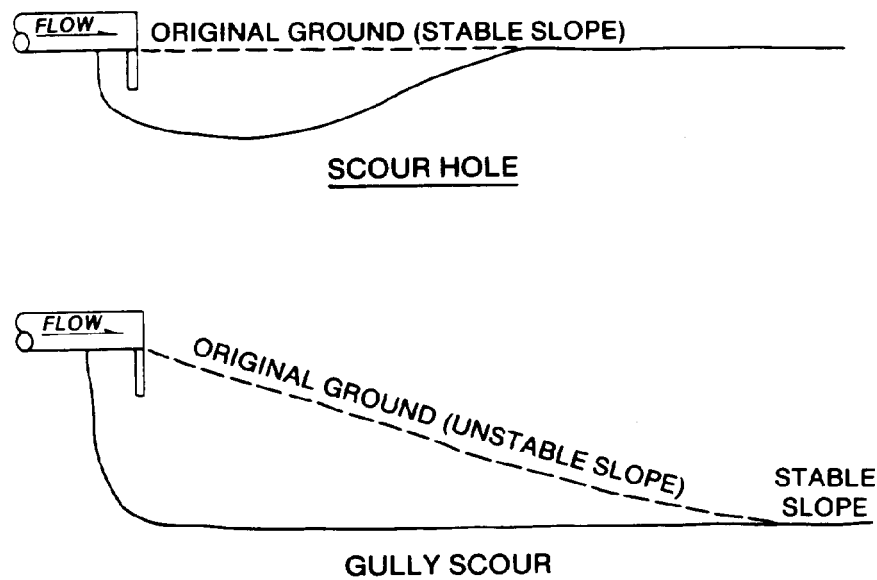


Figure 5-2. Types of scour at storm-drain and culvert outlets.

a. Gully scour is to be expected when the Froude number of flow in the channel exceeds that required for stability. It begins at a control point downstream where the channel is stable and progresses upstream. If sufficient differential in elevation exists between the outlet and the section of stable channel, the outlet structure will be completely undermined. The primary cause of gully scour is the practice of siting outlets high, with or without energy dissipators relative to a stable downstream grade in order to reduce quantities of pipe and excavation. Erosion of this type may be extensive, depending upon the location of the stable channel section relative to that of the outlet in both the vertical and downstream directions. To prevent gully erosion, outlets and energy dissipators should

be located at sites where the slope of the downstream channel or drainage basin is naturally moderate enough to remain stable under the anticipated conditions or else it should be controlled by ditch checks, drop structures, and/or other means to a point where a naturally stable slope and cross section exist. Design of stable open channels is discussed later in this manual.

b. A scour hole or localized erosion can occur downstream of an outlet even if the downstream channel is stable. The severity of damage to be anticipated depends upon the conditions existing or created at the outlet. In many situations, flow conditions can produce scour resulting in embankment erosion as well as structural damage to the apron, endwall, and culvert.

c. Empirical equations have been developed for estimating the extent of the anticipated scour hole in sand, based on knowledge of the design discharge, the culvert diameter, and the duration and Froude number of the design flow at the culvert outlet. However, the relationship between the Froude number of flow at the culvert outlet and a discharge parameter, or $Q/D_o^{5/2}$, can be calculated for any shape of outlet, and this discharge parameter is just as representative of flow conditions as is the Froude number. The relationship between the two parameters, for partial and full pipe flow in square culverts, is shown in figure 5-3. Terms are defined in appendix E. Since the discharge parameter is easier to calculate and is suitable for application purposes, the original data were reanalyzed

in terms of discharge parameter for estimating the extent of localized scour to be anticipated downstream of culvert and storm drain outlets. The equations for the maximum depth, width, length, and volume of scour and comparisons of predicted and observed values are shown in figures 5-4 through 5-7. Minimum and maximum tailwater depths are defined as those less than $0.5D_o$ and equal to or great than $0.5D_o$, respectively. Dimensionless profiles along the center lines of the scour holes to be anticipated with minimum and maximum tailwaters are presented in figures 5-8 and 5-9. Dimensionless cross sections of the scour hole at a distance of 0.4 of the maximum length of scour downstream of the culvert outlet for all tailwater conditions are also shown in figures 5-8 and 5-9.

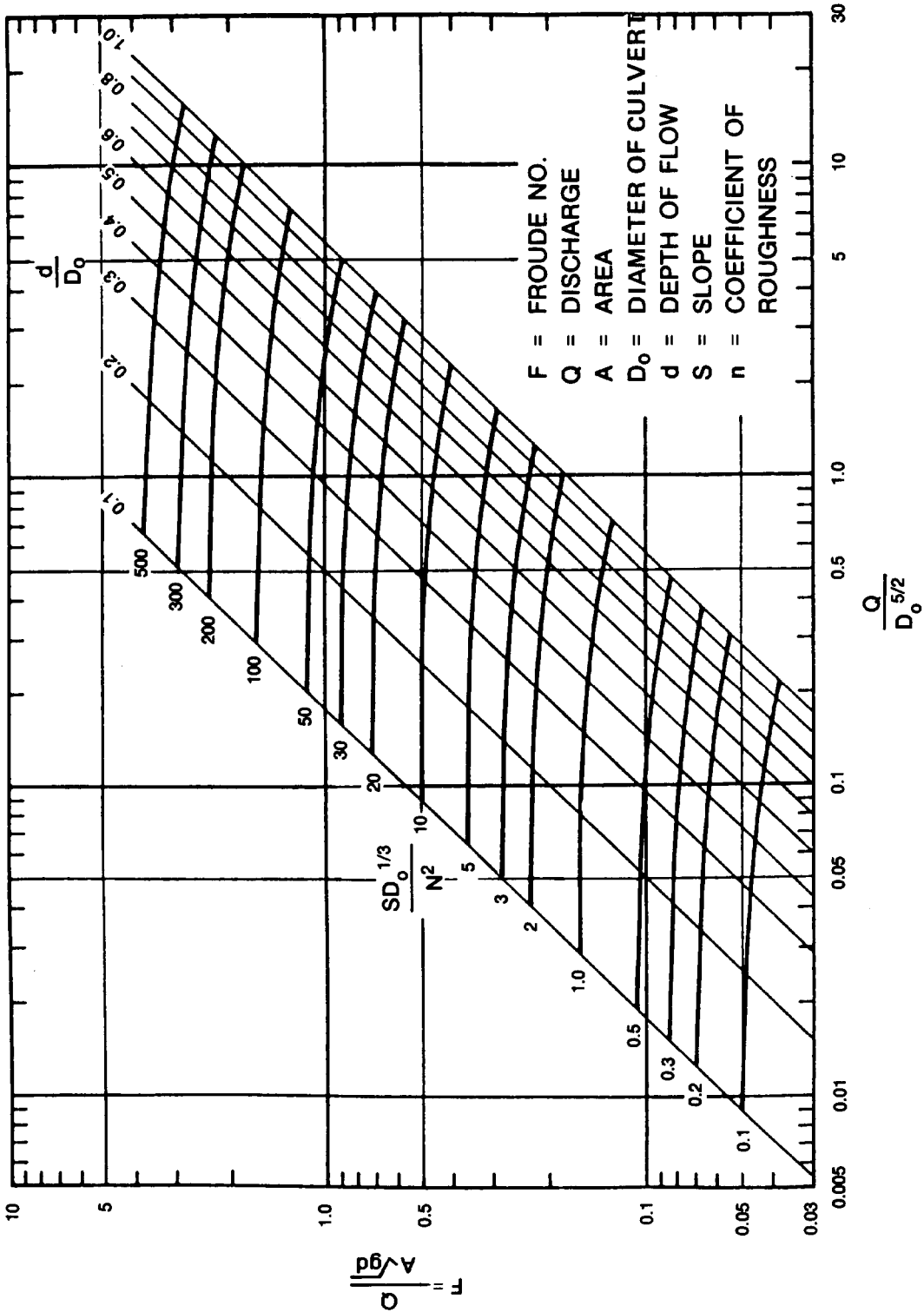


Figure 5-3. Square culvert-Froude number.

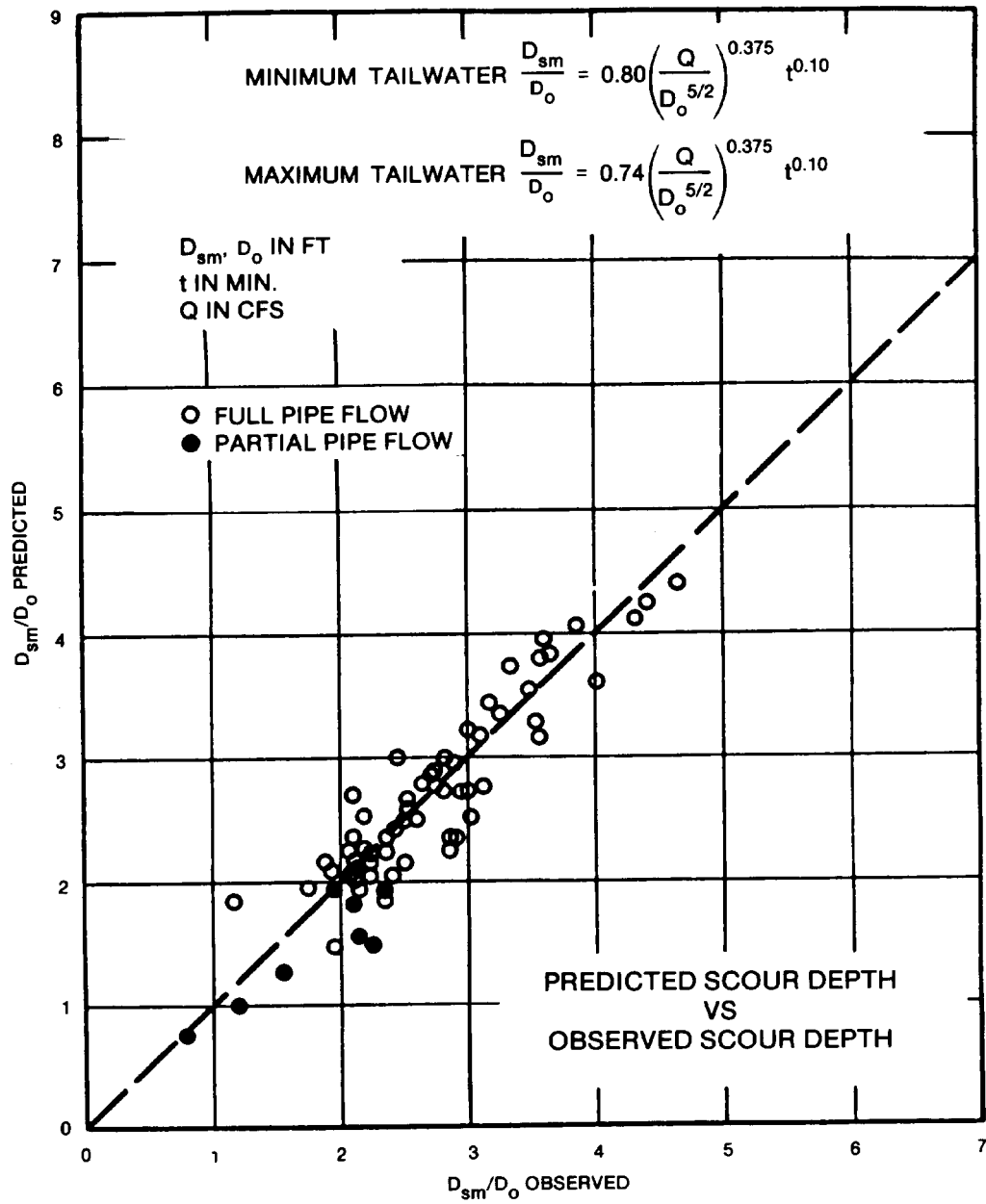


Figure 5-4. Predicted scour depth versus observed scour depth.

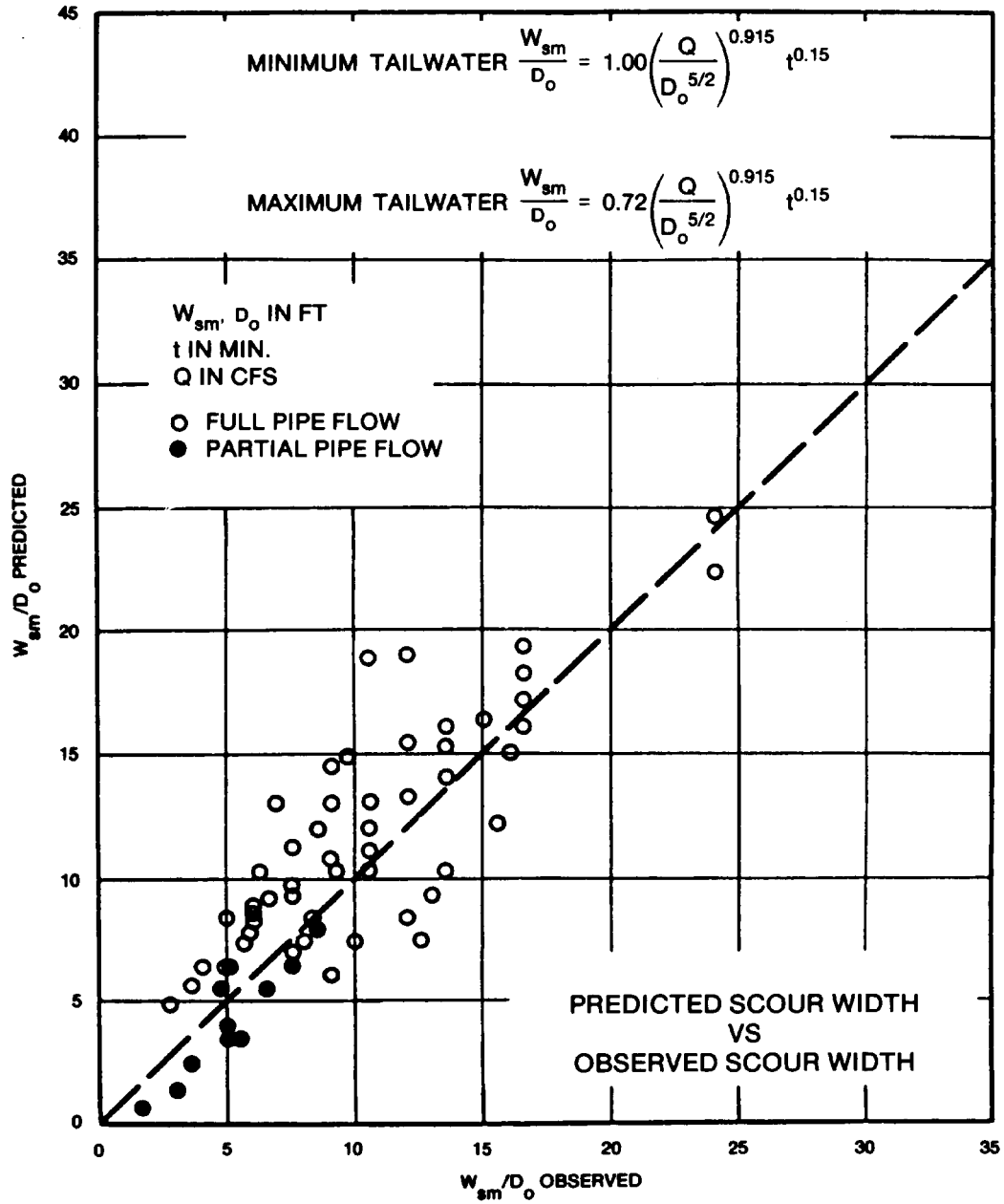


Figure 5-5. Predicted scour width versus observed scour width.

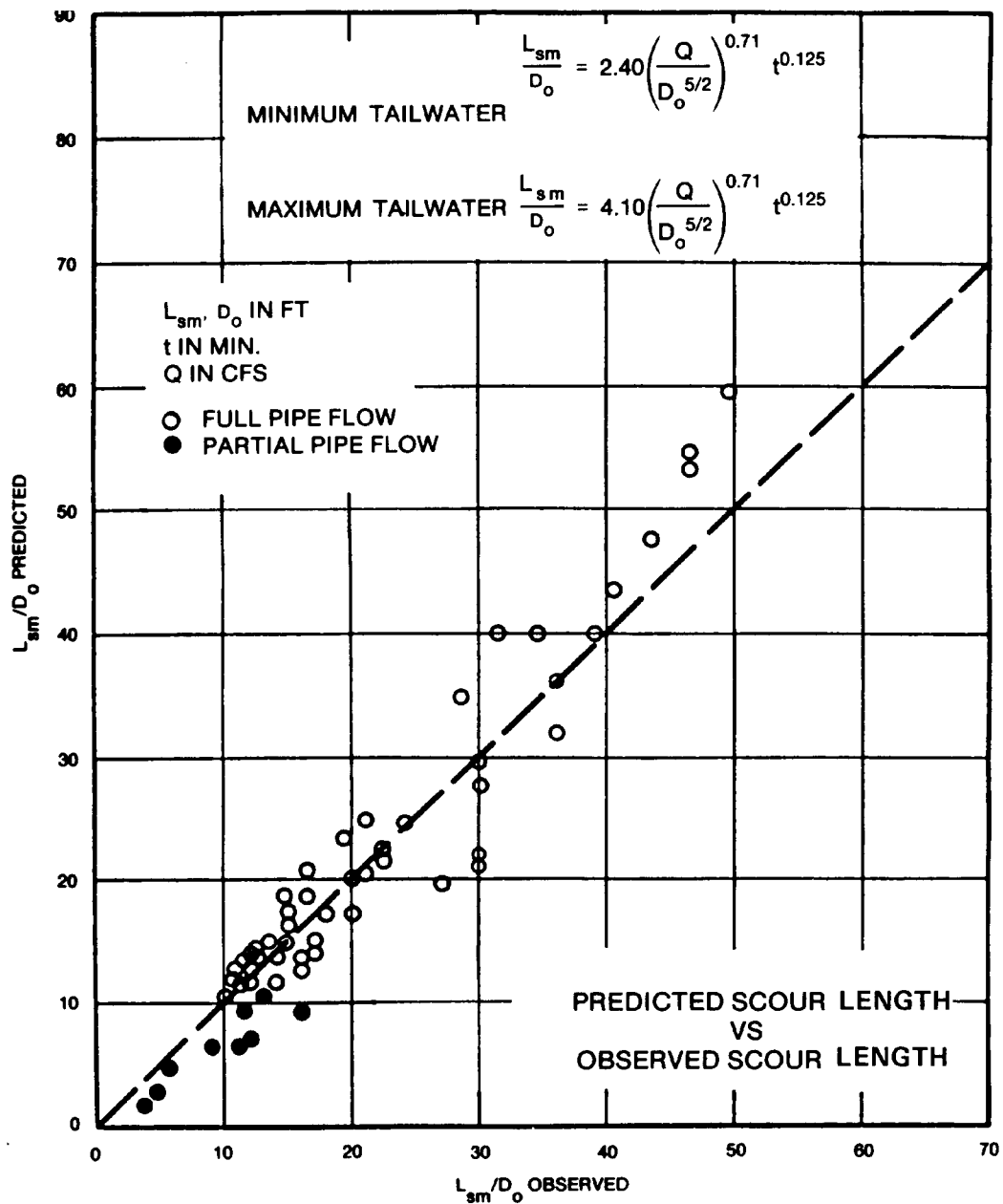


Figure 5-6. Predicted scour length versus observed scour length.

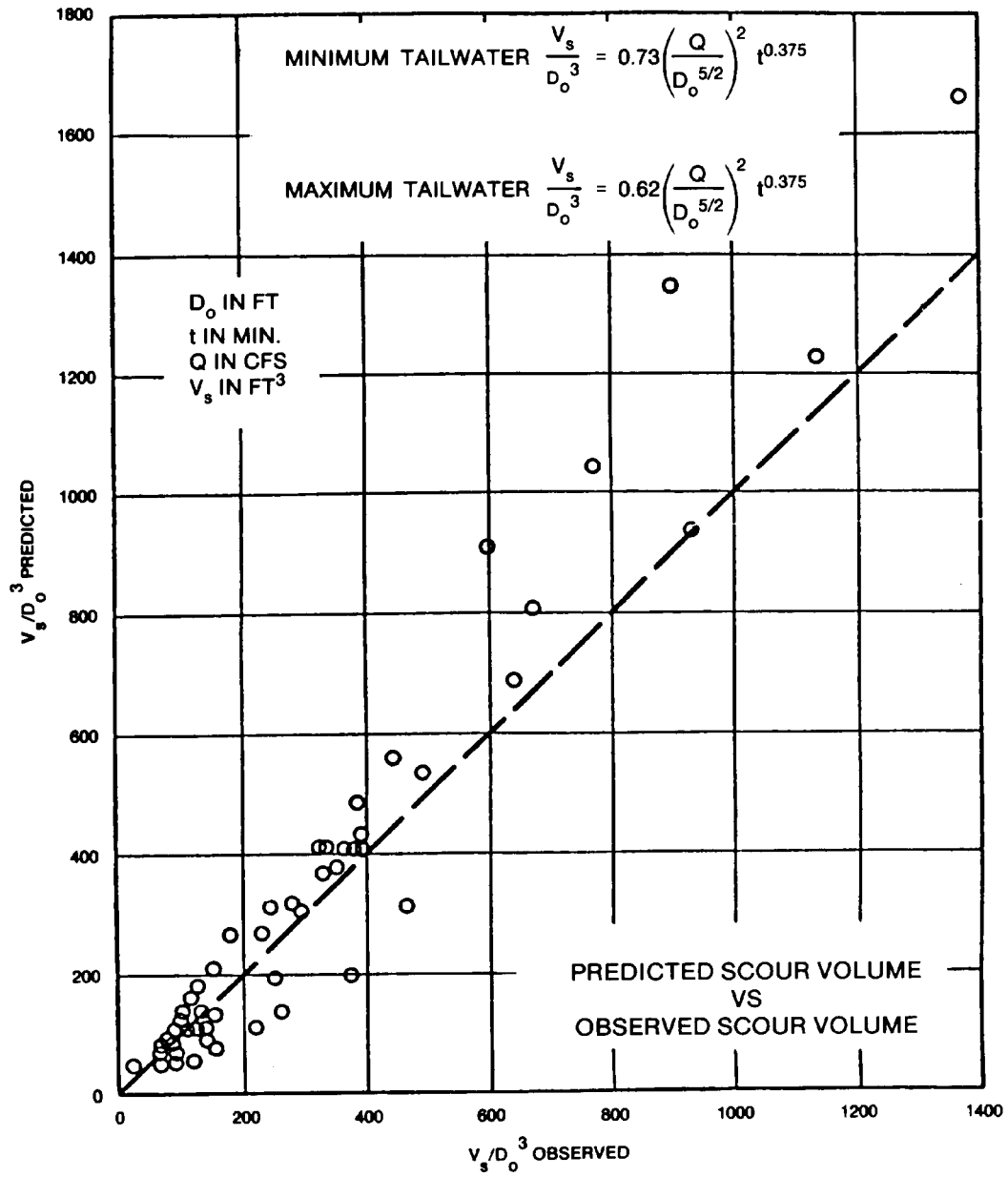


Figure 5-7. Predicted scour volume versus observed scour volume.

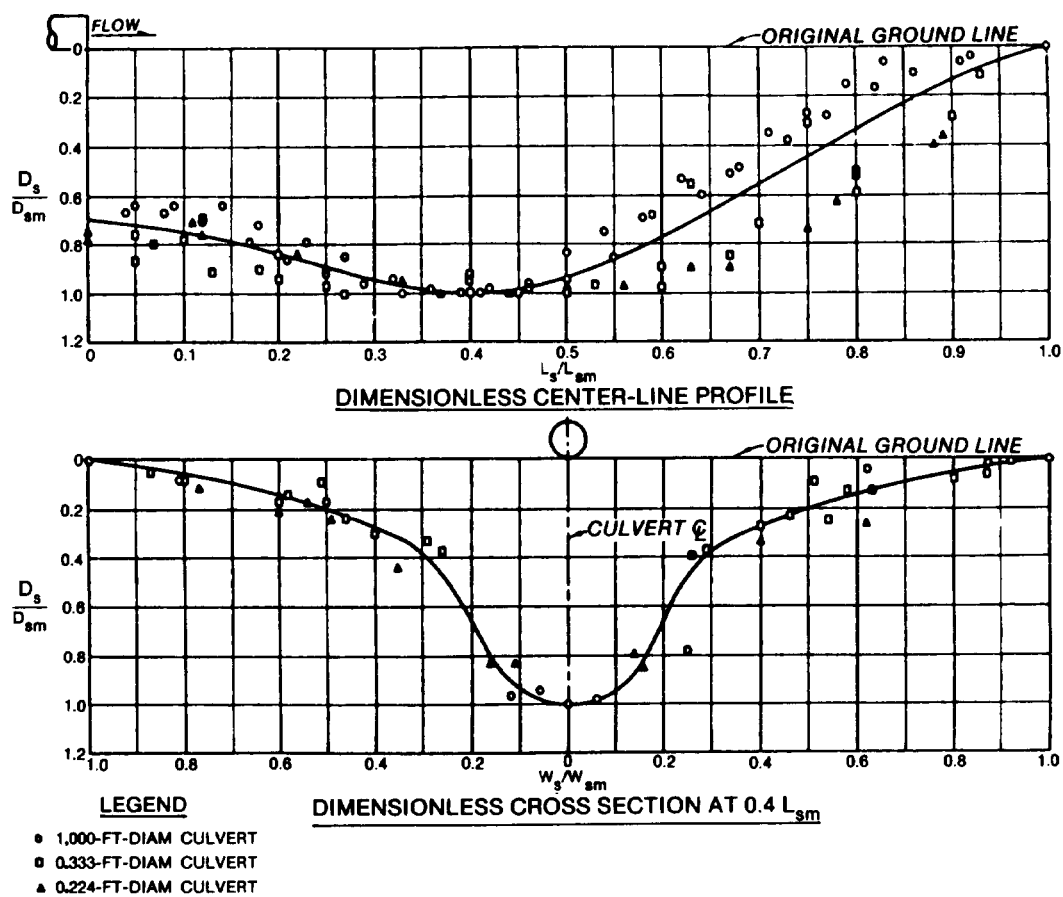


Figure 5-8. Dimensionless scour hole geometry for minimum tailwater.

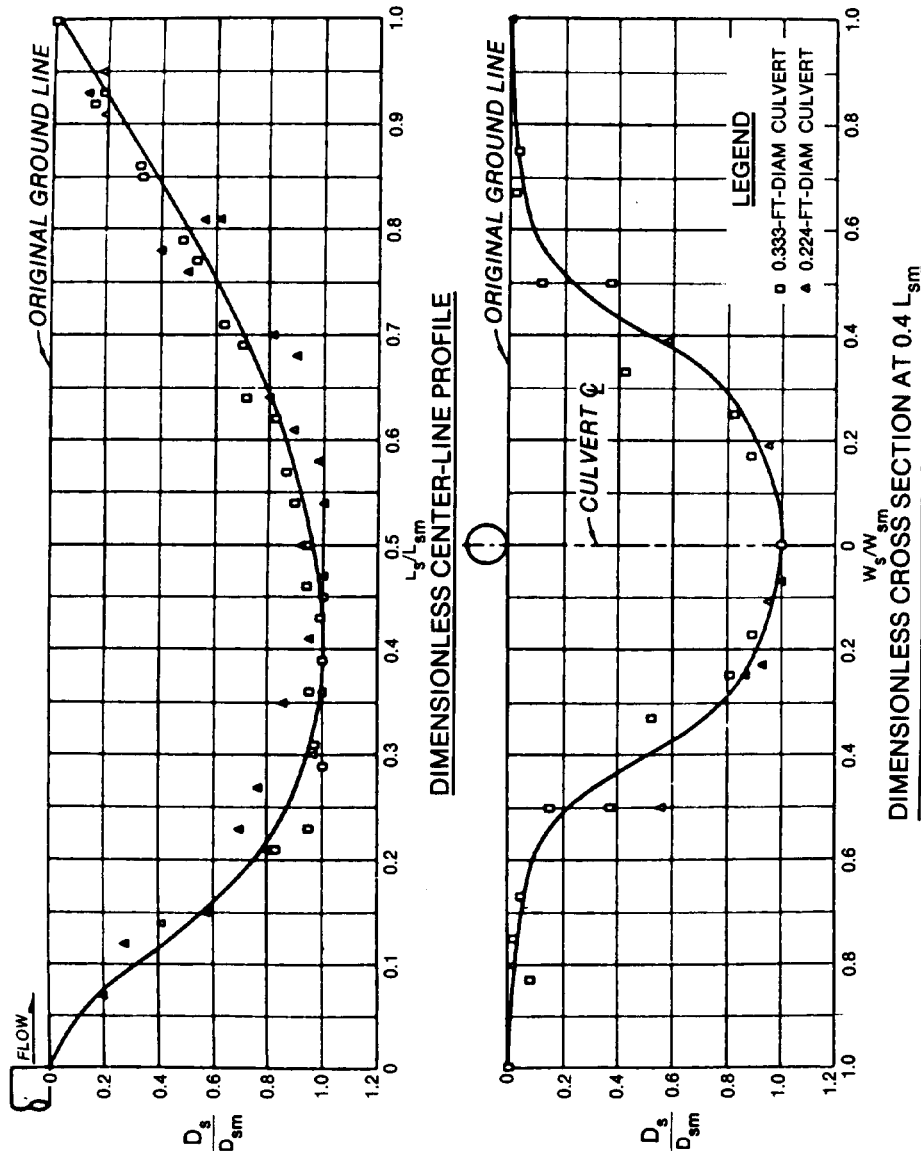


Figure 5-9. Dimensionless scour hole geometry for maximum tailwater.

5-4. Erosion control at outlet.

There are various methods of preventing scour and erosion at outlets and protecting the structure from undermining. Some of these methods will be discussed in subsequent paragraphs.

a. In some situations placement of riprap at the end of the outlet may be sufficient to protect the structure. The average size of stone (d_{50}) and configuration of a horizontal blanket of riprap at outlet invert elevation required to control or prevent localized scour downstream of an outlet can be

estimated using the information in figures 5-10 to 5-12. For a given design discharge, culvert dimensions, and tailwater depth relative to the outlet invert, the minimum average size of stone (d_{50}) for a horizontal blanket of protection can be determined using data in figure 5-10. The length of stone protection (LSP) can be determined by the relations shown in figure 5-11. The variables are defined in appendix E, and the recommended configuration of the blanket is shown in figure 5-12.

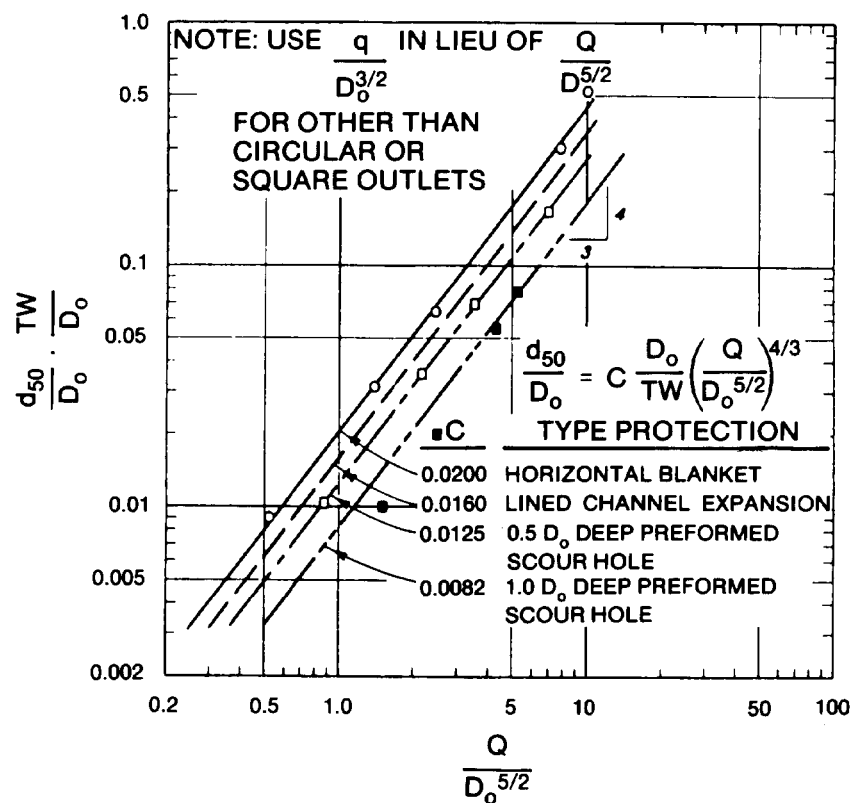


Figure 5-10. Recommended size of protective stone.

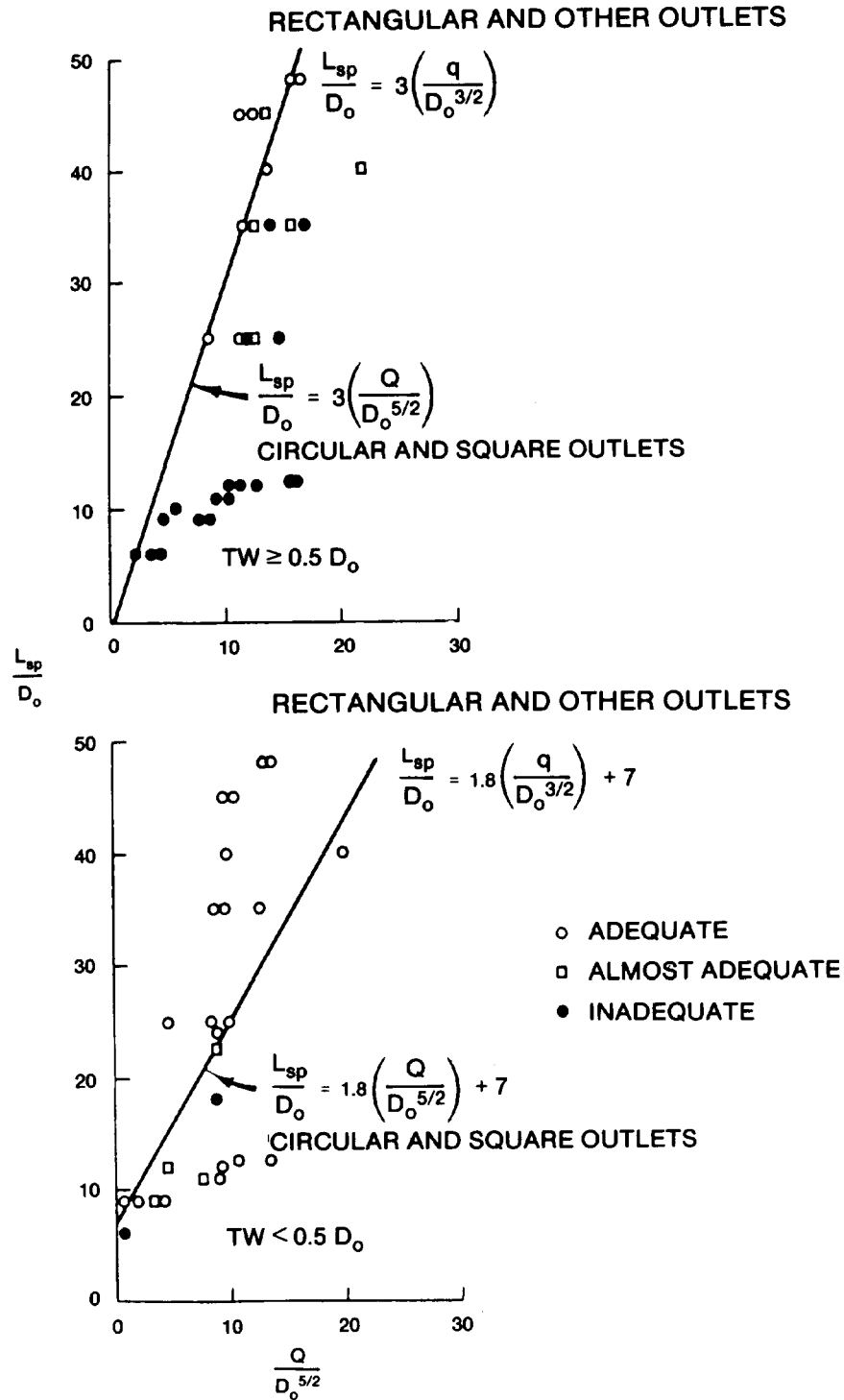


Figure 5-11. Length of stone protection, horizontal blanket.

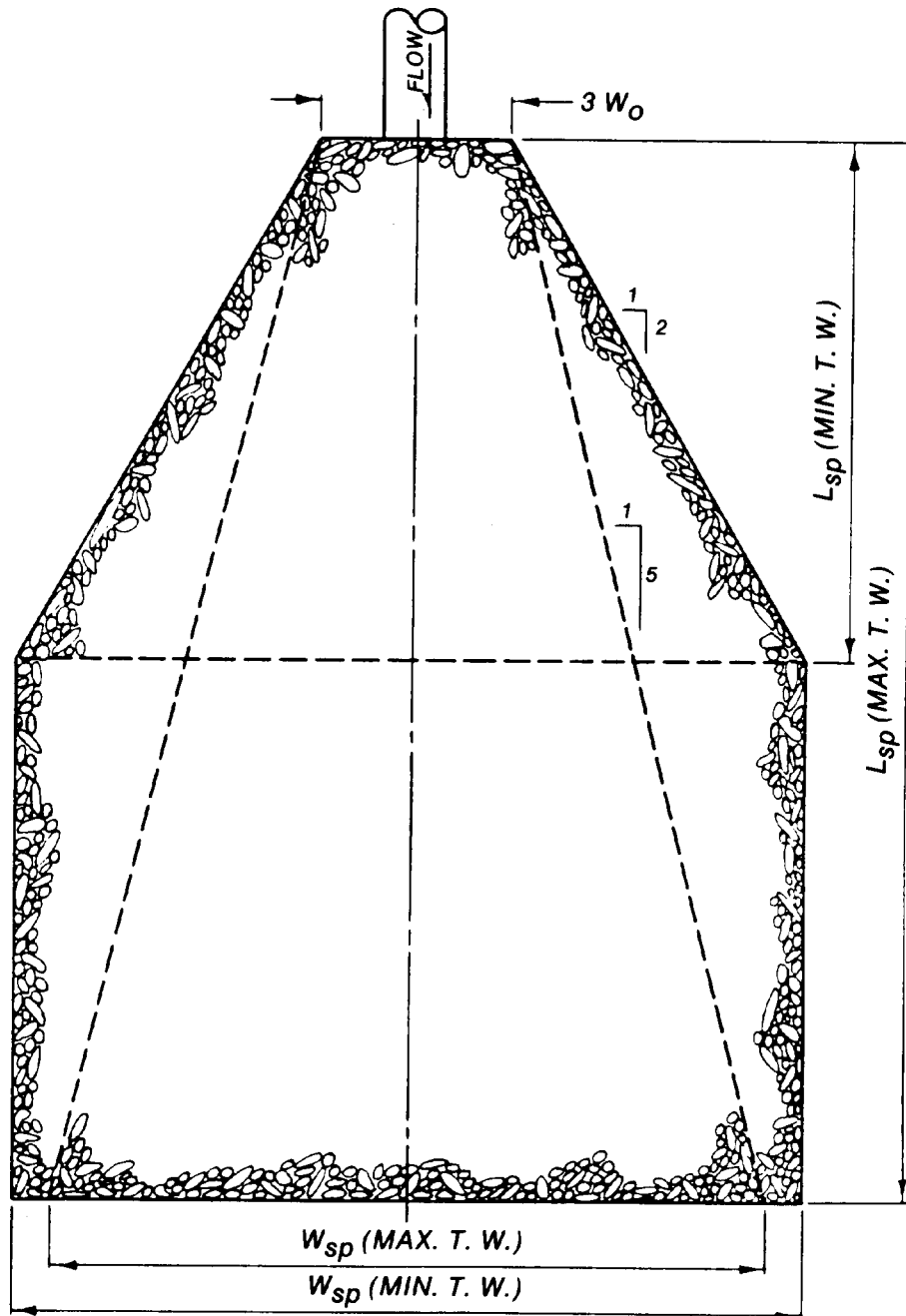


Figure 5-12. Recommended configuration of riprap blanket subject to minimum and maximum tailwaters.

b. The relative advantage of providing both vertical and lateral expansion downstream of an outlet to permit dissipation of excess kinetic energy in turbulence, rather than direct attack of the boundaries, is shown in figure 5-10. Figure 5-10 indicates that the required size of stone may be

reduced considerably if a riprap-lined, preformed scour hole is provided, instead of a horizontal blanket at an elevation essentially the same as the outlet invert. Details of a scheme of riprap protection termed "performed scour hole lined with riprap" are shown in figure 5-13.

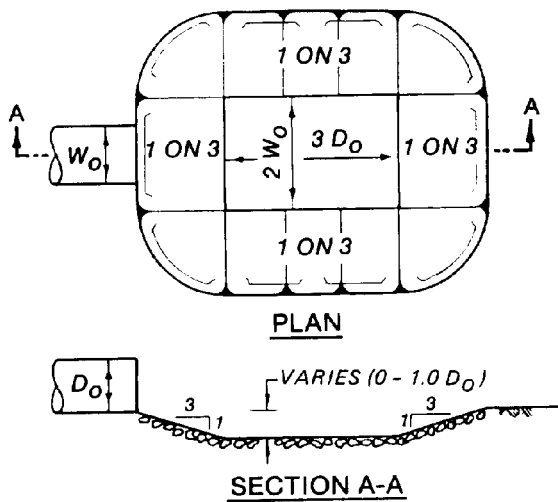


Figure 5-13. Preformed scour hole.

c. Three ways in which riprap can fail are movement of the individual stones by a combination of

velocity and turbulence, movement of the natural bed material through the riprap resulting in slumping of the blanket, and undercutting and raveling of the riprap by scour at the end of the blanket. Therefore, in design, consideration must be given to selection of an adequate size stone, use of an adequately graded riprap or provision of a filter blanket, and proper treatment of the end of the blanket.

d. Expanding and lining the channel downstream from a square or rectangular outlet for erosion control can be with either sack revetment or cellular blocks as well as rock riprap, as placed shown in figure 5-14. The conditions of discharge and tailwater required to displace sack revetment with length, width, and thickness of 2, 1.5, and 0.33 feet, respectively (weight 120 pounds); cellular blocks, 0.66 by 0.66 foot and 0.33 foot thick (weight 14 pounds); or riprap with a given thickness are shown in figure 5-15. The effectiveness of the lined channel expansion relative to the other schemes of riprap protection described previously is shown in figure 5-10.

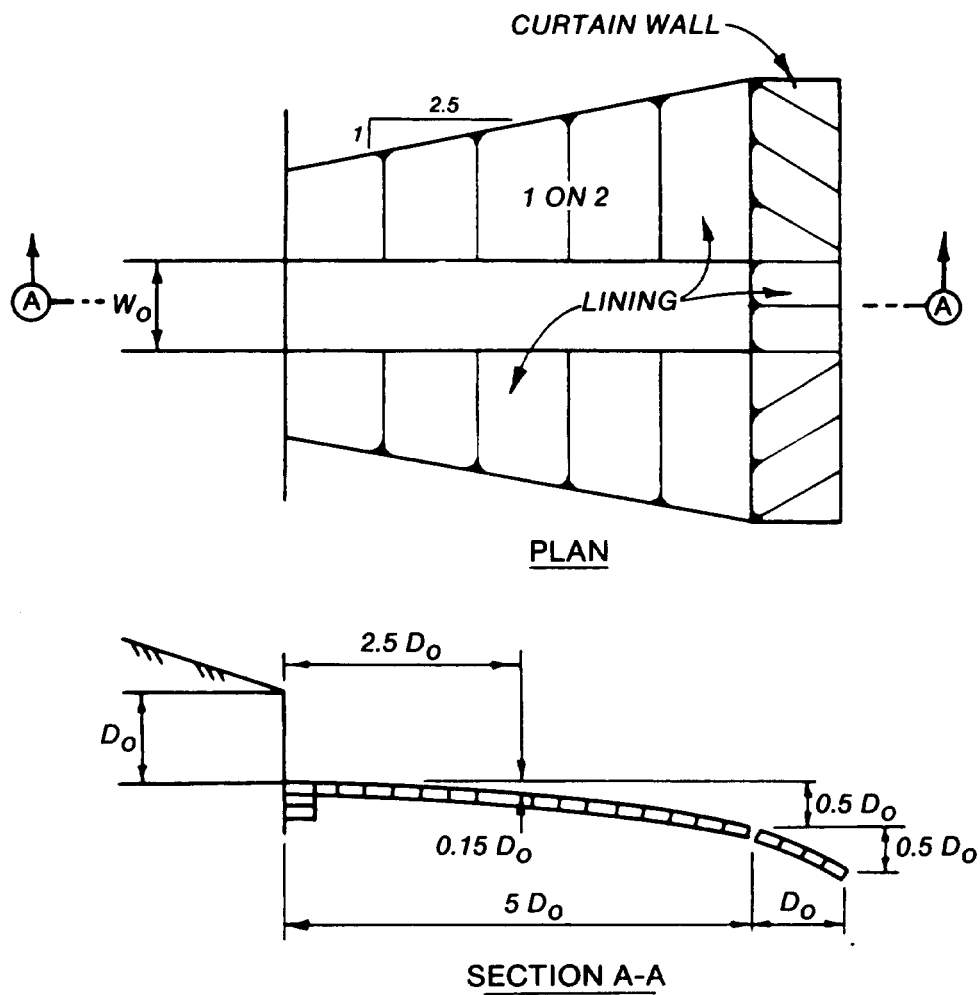


Figure 5-14. Culvert outlet erosion protection, lined channel expansions.

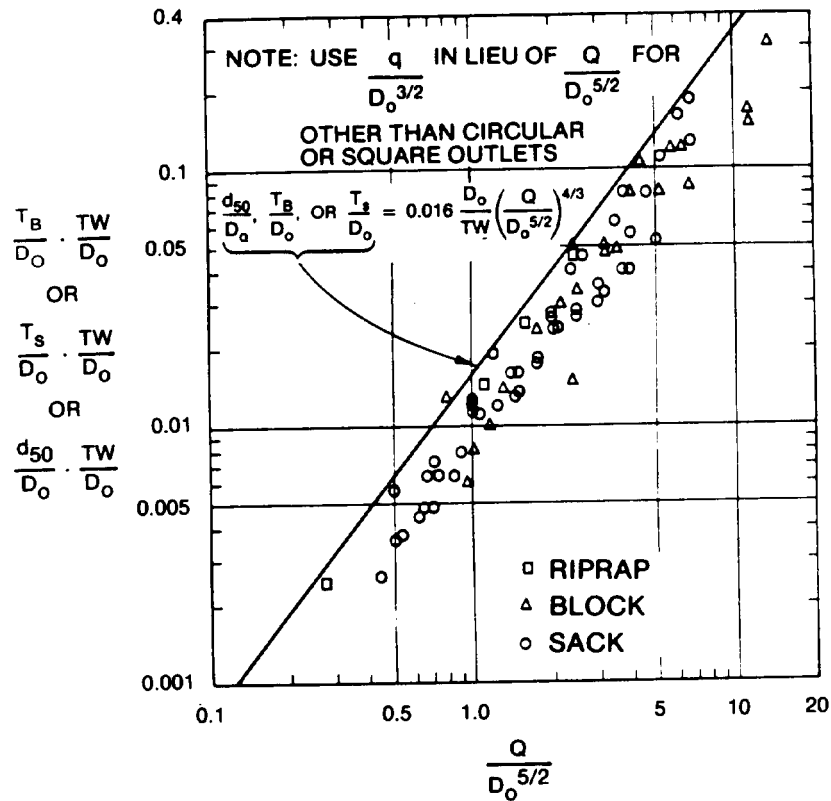


Figure 5-15. Maximum permissible discharge for lined channel expansions.

e. The maximum discharge parameters, $Q/D_o^{5/2}$ or $q/D_o^{3/2}$, of various schemes of protection can be calculated based on the above information; comparisons relative to the cost of each type of protection can then be made to determine the most practical design for providing effective drainage and erosion control facilities for a given site. There will be conditions where the design discharge and economical size of conduit will result in a value of the discharge parameter greater than the maximum value permissible thus requiring some form of energy dissipator.

f. The simplest form of energy dissipator is the flared outlet transition. Protection is provided to the local area covered by the apron, and a portion of the kinetic energy of flow is reduced or converted to potential energy by hydraulic resistance provided by the apron. A typical flared outlet transition is shown in figure 5-16. The flare angle of the walls should be 1 on 8. The length of transition needed for a given discharge conduit size and tailwater situation with the apron at the same elevation as the outlet invert ($H = 0$) can be calculated by the following equations.

$$\frac{L}{D_o} = 0.30 \left(\frac{D_o}{TW} \right)^2 \left(\frac{Q}{D_o^{5/2}} \right)^{2.5} (TW/D_o)^{1/3} \quad \text{Circular and square outlets} \quad (\text{eq 5-1})$$

$$\frac{L}{D_o} = 0.30 \left(\frac{D_o}{TW} \right)^2 \left(\frac{q}{D_o^{3/2}} \right)^{2.5} (TW/D_o)^{1/3} \quad \text{Rectangular and other shaped outlets} \quad (\text{eq 5-2})$$

Recessing the apron and providing an end sill will not significantly improve energy dissipation.

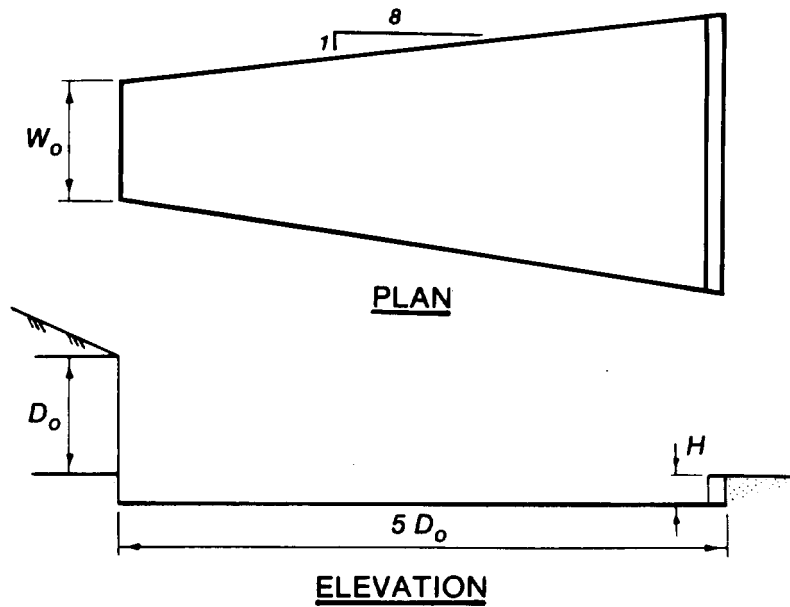
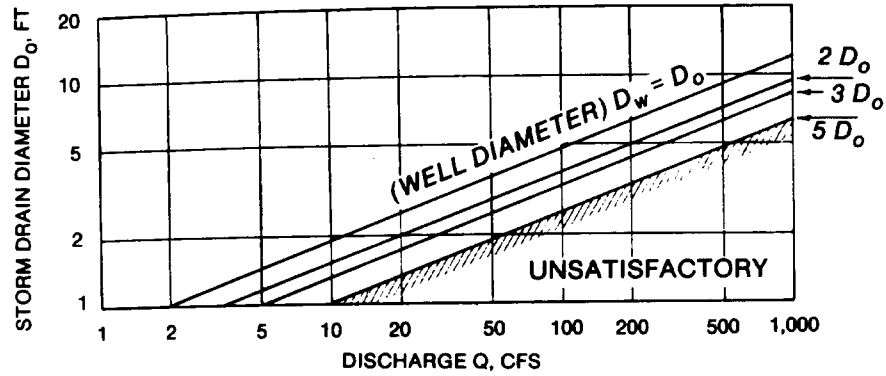


Figure 5-16. Flared outlet transition.

g. The flared transition is satisfactory only for low values of $Q/D_o^{5/2}$ or $q/D_o^{3/2}$ as will be found at culvert outlets. With higher values, however, as will be experienced at storm drain outlets, other types of energy dissipators will be required. Design criteria for three types of laboratory tested energy

dissipators are presented in figures 5-17 to 5-19. Each type has advantages and limitations. Selection of the optimum type and size is dependent upon local tailwater conditions, maximum expected discharge, and economic considerations.



BASIC EQUATION

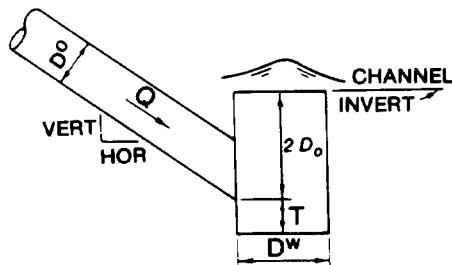
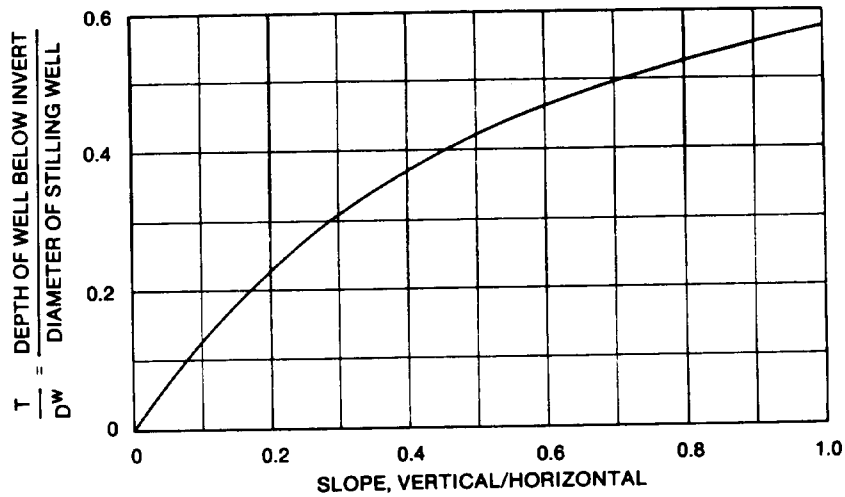
$$\frac{D_w}{D_o} = 0.53 \left(\frac{Q}{D_o^{5/2}} \right) \text{ FOR } \frac{Q}{D_o^{5/2}} \leq 10$$

WHERE:

D_w = STILLING WELL DIAMETER, FT

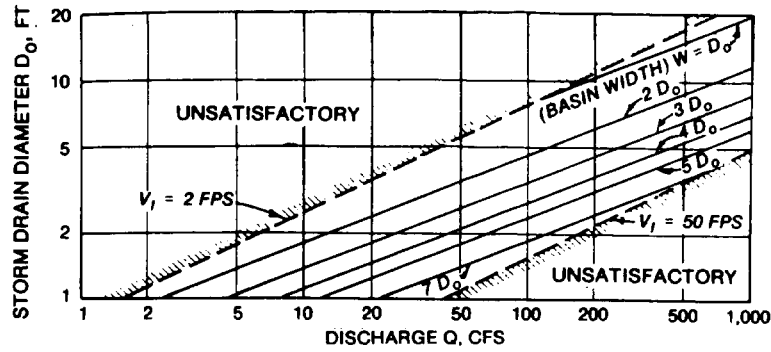
D_o = DRAIN DIAMETER, FT

Q = DESIGN DISCHARGE, CFS



ELEVATION

Figure 5-17. Stilling well.

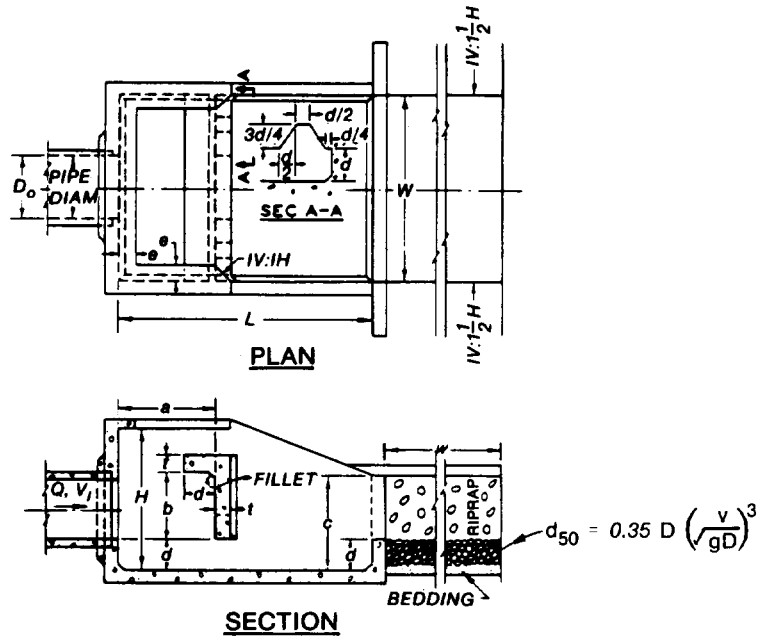


BASIC EQUATION

$$\frac{W}{D_o} = 1.3 \left(\frac{Q}{D_o^{5/2}} \right)^{0.55} \text{ FOR } \frac{Q}{D_o^{5/2}} \leq 21$$

WHERE:

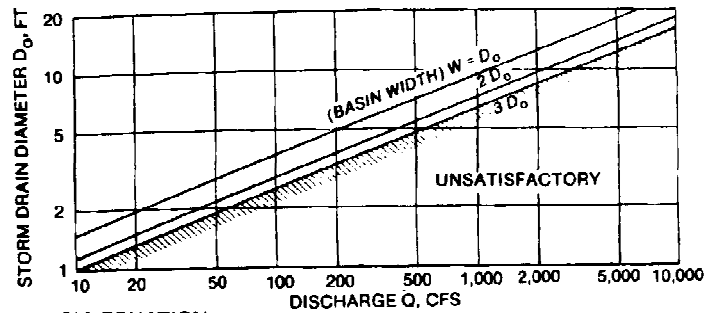
W = BASIN WIDTH, FT
 D_o = DRAIN DIAMETER, FT
 Q = DESIGN DISCHARGE, CFS
 V_i = PIPE VELOCITY, FPS



STILLING BASIN DESIGN

$$\begin{aligned} H &= \frac{3}{4} (W) & c &= \frac{1}{2} (W) \\ L &= \frac{4}{3} (W) & d &= \frac{1}{6} (W) \\ a &= \frac{1}{2} (W) & e &= \frac{1}{12} (W) \\ b &= \frac{3}{8} (W) & t &= \frac{1}{12} (W), \text{ SUGGESTED MINIMUM} \end{aligned}$$

Figure 5-18. US Bureau of Reclamation impact basin.



BASIC EQUATION

$$\frac{W}{D_0} = 0.3 \frac{Q}{D_0^{5/2}} \quad \text{FOR } \frac{Q}{D_0^{5/2}} \leq 9.5$$

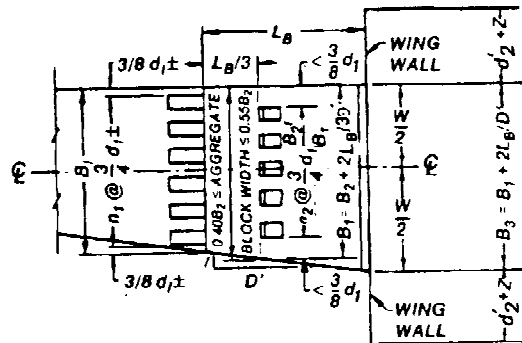
WHERE:

W = END SILL LENGTH, FT

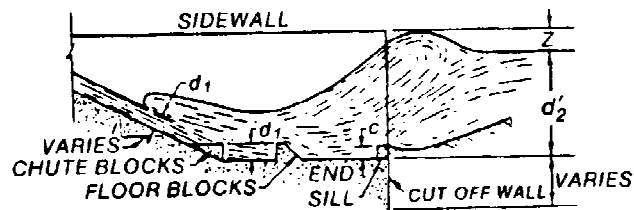
D^o = DRAIN DIAMETER, FT

Q = DESIGN DISCHARGE, CFS

STILLING BASIN HALF-PLAN



TRAPEZOIDAL STILLING BASIN HALF-PLAN



CENTER-LINE SECTION

DESIGN EQUATIONS

$$F = \frac{V_1^2}{g d_1} \quad (1) \quad L_B = \frac{4.5 d_2}{F^{0.38}} \quad (4)$$

$$d_2 = \frac{d_1}{2}(-1 + \sqrt{8F + 1}) \quad (2) \quad Z = \frac{d_2}{3} \quad (5)$$

$$E = 3 \text{ TO } 30 \quad d_1 = (1.10 - E/120)d_2 \quad (3a) \quad c = 0.07d_2 \quad (8)$$

$$F = 3 \text{ TO } 30 \quad d_1 = (1.10 - F/120)d_2 \quad (3a) \quad c = 0.07d_2 \quad (b)$$

\mathbb{G}_2 (1970-1975), $\mathbb{Z} = 1$

$$F = 30 \text{ TO } 120 \quad d_2 = 0.85d_3 \quad (3b)$$

100-70-000 100-70-000-1 (7.0)

Figure 5-19. Saint Anthony Falls stilling basin.

h. The stilling well shown in figure 5-17 consists of a vertical section of circular pipe affixed to the outlet end of a storm sewer. The recommended depth of the well below the invert of the incoming

pipe is dependent on the slope and diameter of the incoming pipe and can be determined from the plot in figure 5-17. The recommended height above the invert of the incoming pipe is two times the

diameter of the incoming pipe. The required well diameter can be determined from the equation in figure 5-17. The top of the well should be located at the elevation of the invert of a stable channel or drainage basin. The area adjacent to the well may be protected by riprap or paving. Energy dissipation is accomplished without the necessity of maintaining a specified tailwater depth in the vicinity of the outlet. Use of the stilling well is not recommended with $Q/D_o^{5/2}$ greater than 10.

i. The US Bureau of Reclamation (USBR) impact energy dissipator shown in figure 5-18 is an efficient stilling device even with deficient tailwater. Energy dissipation is accomplished by the impact of the entering jet on the vertically hanging baffle and by the eddies that are formed following impact on the baffle. Excessive tailwater causes flow over the top of the baffle and should be avoided. The basin width required for good energy dissipation for a given storm drain diameter and discharge can be calculated from the information in figure 5-18. The other dimensions of energy dissipator are a function of the basin width as shown in figure 5-18. This basin can be used with $Q/D_o^{5/2}$ ratios up to 21.

j. The Saint Anthony Falls (SAF) stilling basin shown in figure 5-19 is a hydraulic jump energy dissipator. To function satisfactorily this basin must have sufficient tailwater to cause a hydraulic jump to form. Design equations for determining the dimensions of the structure in terms of the square of the Froude number of flow entering the dissipator are shown in this figure. Figure 5-20 is a design chart based on these equations. The width of basin required for good energy dissipation can be calculated from the equation in figure 5-19. Tests used to develop this equation were limited to basin widths of three times the diameter of the outlet. But, other model tests indicate that this equation also applies to ratios greater than the maximum shown in figure 5-19. However, outlet portal velocities exceeding 60 feet per second are not recommended for design containing chute blocks. Parallel basin sidewalls are recommended for best performance. Transition sidewalls from the outlet to the basin should not flare more than 1 on 8.

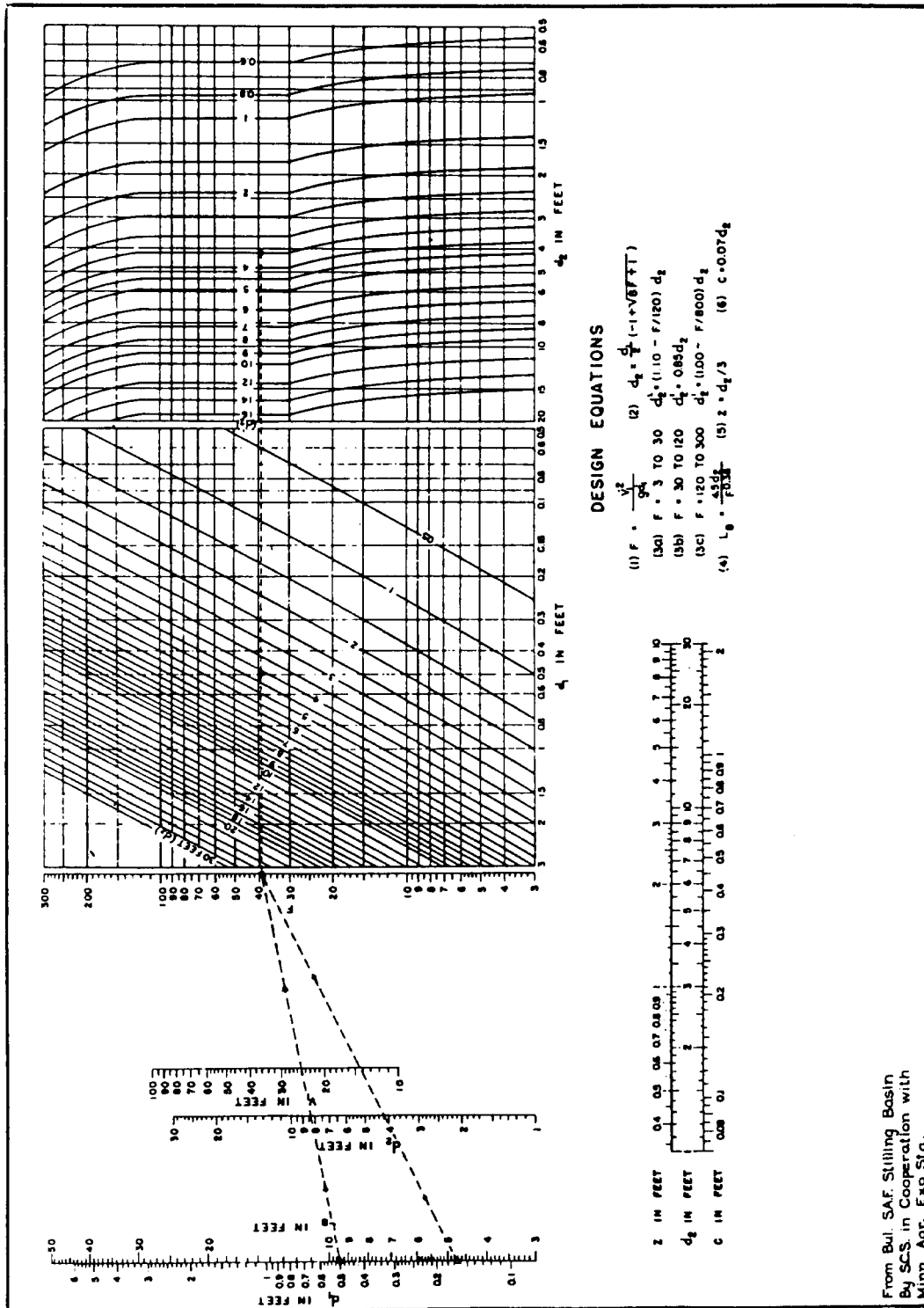


Figure 5-20. Design chart for SAF stilling basin.

k. Riprap Will be required downstream from the above energy dissipators. The size of the stone can be estimated by the following equation.

$$d_{s0} = D \left(\frac{V}{\sqrt{gD}} \right)^3 \quad \text{or } F = \left(d_{s0}/D \right)^{1/3} \quad (\text{eq 5-3})$$

This equation is also to be used for riprap subject to direct attack or adjacent to hydraulic structures such as inlets, confluences, and energy dissipators, where turbulence levels are high. The riprap should extend downstream for a distance approximately 10 times the theoretical depth of flow required for a hydraulic jump.

l. Smaller riprap sizes can be used to control channel erosion. Equation 5-4 is to be used for riprap on the banks of a straight channel where flows are relatively quiet and parallel to the banks.

Trapezoidal channels

$$d_{s0} = 0.35 D \left(\frac{V}{\sqrt{gD}} \right)^3 \quad \text{or } F = 1.42 \left(d_{s0}/D \right)^{1/3} \quad (\text{eq 5-4})$$

Equation 5-5 is to be used for riprap at the outlets of pipes or culverts where no preformed scour holes are made.

Wide channel bottom or horizontal scour hole

$$d_{s0} = 0.15 D \left(\frac{V}{\sqrt{gD}} \right)^3 \quad \text{or } F = 1.88 \left(d_{s0}/D \right)^{1/3} \quad (\text{eq 5-5})$$

1/2 D deep scour hole

$$d_{s0} = 0.09 D \left(\frac{V}{\sqrt{gD}} \right)^3 \quad \text{or } F = 2.23 \left(d_{s0}/D \right)^{1/3} \quad (\text{eq 5-6})$$

D deep scour hole

$$d_{s0} = 0.055 D \left(\frac{V}{\sqrt{gD}} \right)^3 \quad \text{or } F = 2.63 \left(d_{s0}/D \right)^{1/3} \quad (\text{eq 5-7})$$

These relationships are shown in figures 5-21 and 5-22.

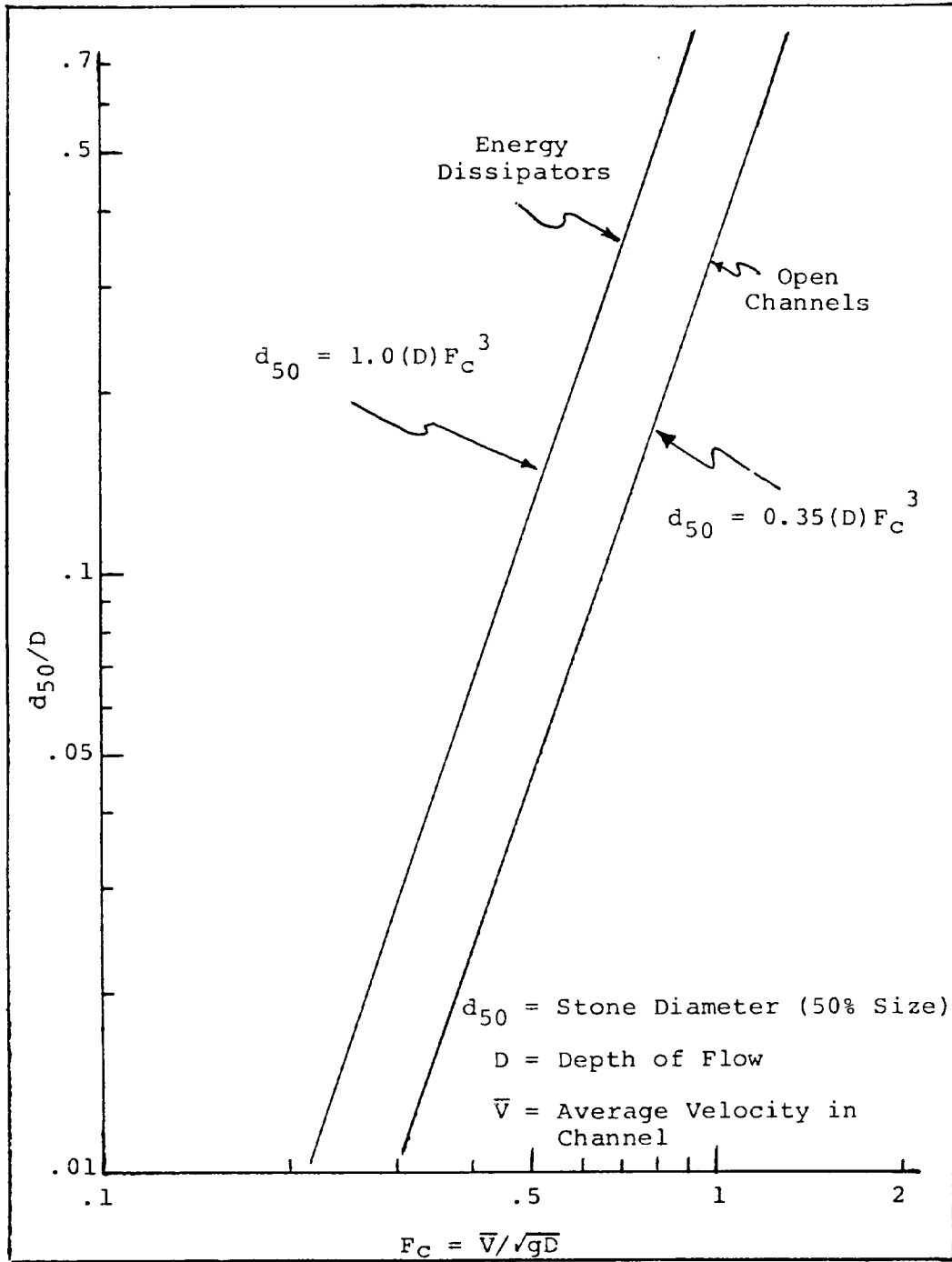


Figure 5-21. Recommended riprap sizes.

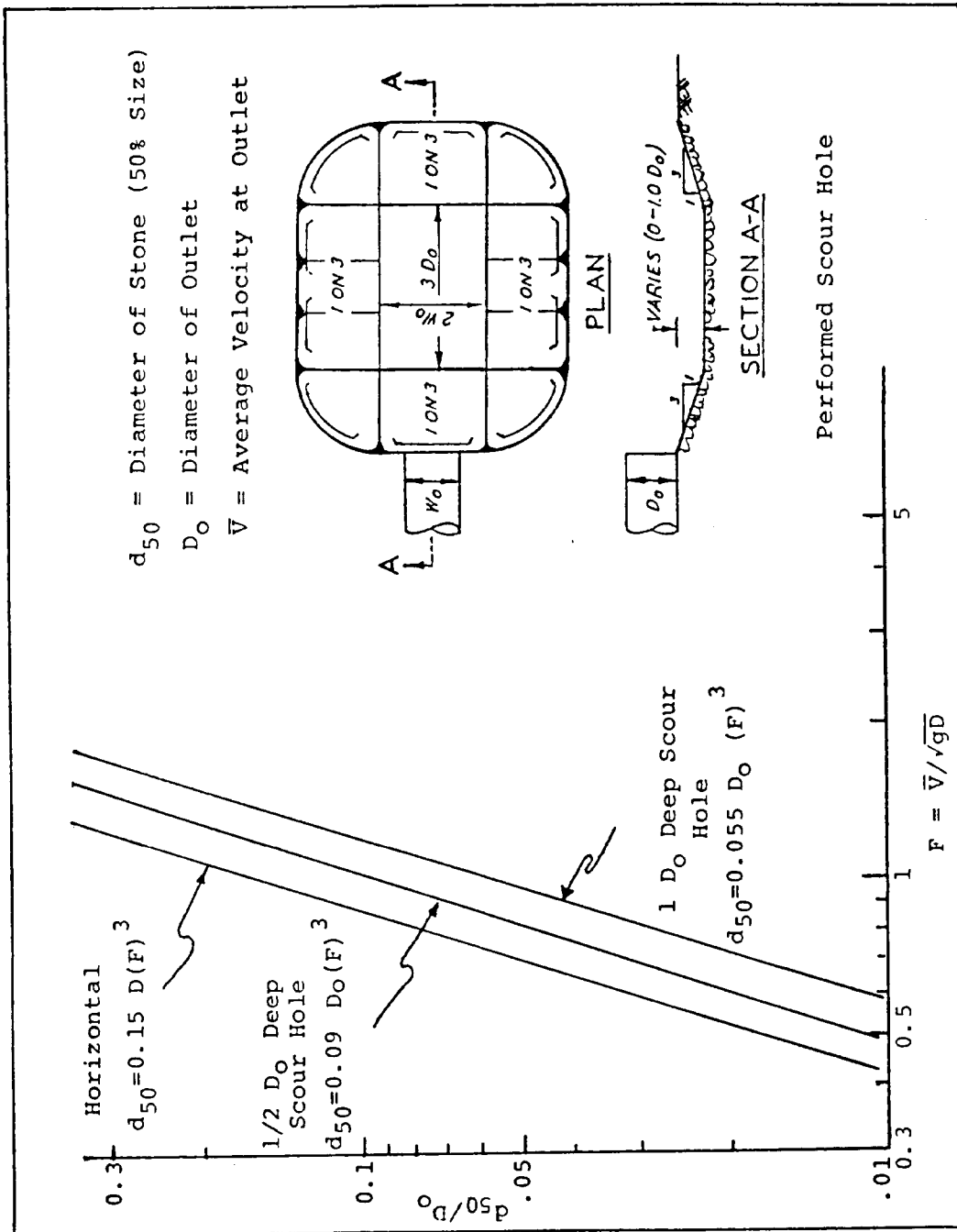


Figure 5-22. Scour hole riprap sizes.

m. Examples of recommended application to estimate the extent of scour in a cohesionless soil and several alternate schemes of protection required to prevent local scour downstream from a circular and rectangular outlet are shown in appendix C.

n. User-friendly computer programs are available to assist the designer with many of the design problems discussed in this chapter (Conversationally Oriented Real-Time Program Generating System (CORPS)). These programs are available from CEWES-LIB, U.S. Army Engineer Waterways Experiment Station, P0 Box 631, Vicksburg, MS 39180-0631.